The plumbing of 2.2° inclined tall building

La verticalisation d'un grand bâtiment incliné 2.2°

C.E.M.Maffei – *Maffei Engenharia, Polytechnic School, University of São Paulo, Brazil*
H.H.S.Gonçalves & P.M.Pimenta – *Polytechnic School, University of São Paulo, Brazil*
C.A.Murakami – *Maffei Engenharia, São Paulo, S.P., Brazil*

ABSTRACT: This paper addresses the straightening up project of a reinforced concrete building complex in Santos, Brazil, which is composed by two towers. At the ending of the year 1998, before the recovery works Tower A was *2.08 m* out of vertical position while Tower B was almost the same. Tower A is *55.0 m* high, *10.50 m* wide and *24.0 m* long, resembling the famous case of the leaning Tower of Pisa, which is *58.0 m* high and has a diameter of *12.5 m* [Leonhardt, 1997]. First, a safety assessment analysis was undertaken. Application of modern computational considering realistic constitutive models for the concrete and steel as well as non-linear geometrical effects, has shown that the strengthening of the structure was urgent, since the buildings could be in risk of structural collapse after ten years. Sixteen excavated large diameter concrete piles were built as the new foundation. They have reached the depth of *55.0 m* and assure an almost settlement-free foundation. A complex beam system was conceived to transfer the load to the new foundation. On January 2001 the building was then lifted by 14 hydraulic jacks to the right vertical position.

RÉSUMÉ: Ce travail aborde la verticalisation d'un ensemble de bâtiments en béton armé à Santos, Brésil. Lénsemble est composé de deux tours. Au terme de l'année 1998 Tour A etait tort *2,08 m* en dehors de la position verticale et Tour B presque la même chose. Tour A a une hauteur de *57,0 m*, une largeur de *10,5* m et une longueur de *24,0* m, ressemblant assez à la célèbre Tour de Pise qui a une hauteur de *58,0* m et un diamètre de *12,5 m* [Leonhardt, 1997.Calcules faites à l'aide de um ordinateur considérant des equations constitutifs réalistes pour le béton et pour l'acier comme des effets géometriques non-linéaires, ont montré que le renforcement de la structure était urgent parce que les bâtiments riquaint de tomber díci 10 ans. Seize trous à grand diamètre ont été creusés et remplis avec du béton dont le rôle être la nouvelle fondation. Ces pilliers ont atteint une profondeur de *55,0 m* et assurent unefondation presque libre de deplacement. Un système complexe de poutres a été construit pour transférer la charge à la nouvelle fondation. En Janvier 2001, le bâtiment a été souleve par 14 crics hydrauliques à la position verticale.

1 INTRODUCTION

The underground of the city of Santos, Brazil, is well known for its effects on tall buildings supported by shallow foundations. The city, 60 km far from São Paulo, has the largest Brazilian harbor. During the fifties and sixties several tall buildings were constructed along the beach shore. Today there are about 100 buildings leaning on a progressively manner due to consolidation settlements some of them have their structural stability threatened.

The building foundations were designed in footings laid on the superficial sand layer at 1.5m to 2m depth.

This superficial sand layer of 12m thickness is supported by deep soft clay layers (fluvio-lacustrine sediments) with OCR about 1.3, approximately 18m thick. Under this layer there is another sand layer of 3m to 5m and a clay (transitional clays) layer 13m thick with OCR higher than 2. The residual soil is found at the depth of 45 m. The underground profile is shown in figure 1.

Tower A of the "Condomínio Edifício Núncio Malzoni" was built on 1967 on shallow foundations at 2.0 m depth interlocked by rigid beams with 30cm x 150cm. In the same period of the construction of Tower A, a similar building was erected on its left side. The superposition of the pressure bulbs on the deep soft clay layers caused additional settlements making both buildings to lean, one against the other, as it can be seen on figure 9.

The inclination has been monitored by settlement measures of several columns with respect to a bench mark. The measured settlements of the corner columns A1, A3, A30 and A33 (figure 2) since 1972 are displayed in figure 3. Column A1 has settled 26 cm since the beginning of the measurements while column A33 presents the total value 53 cm. Nevertheless the inclination measurement showed that the reai differential settlement between A1 and A33 was 65cm This difference is responsible for the inclination of more than 2.2o observed at the frontal view and 0.6o at the lateral view. Despite of this inclination the structure has not presented any cracking because the building has moved as a rigid body. With exception of a tentative foundation strengthening done in 1978 with the aid of root piles, both columns presented an almost constant settlement velocity of 9mm/year in A1 and 13mm in A33 along the last 20 years! Several modern consolidation theories were examined elsewhere [Gonçalves, 1992] with the purpose of better fitting the experimental data. It was necessary to include viscous effects in the soil constitutive model in order to reproduce the long-term consolidation.

2 SAFETY ASSESSMENT ANALYSIS

A careful study was accomplished, in order to assess the real conditions of structural stability. A modern computational program, named "PORCA" was used in this verification, which was developed at University of São Paulo. This program models the concrete structure as a spatial frame with slabs, considering realistic constitutive models for the concrete and steel, which encompass elasto-plasticity, creep, damage and cracking. Non-linear geometrical effects are taken exactly into account as derived in [Pimenta, 1993].

The complete structure of the building was discretised in over 30000 frame and plate elements. It was assumed a mean settlement plane, which was applied as prescribed displacements after the application of the dead loading. Due to the overturning moment the normal forces on the left columns have increased by 20% over the values in the vertical position. Additional bending moments have strongly changed the stress distribution in columns and beams, as well. These overcharges are equivalent to a

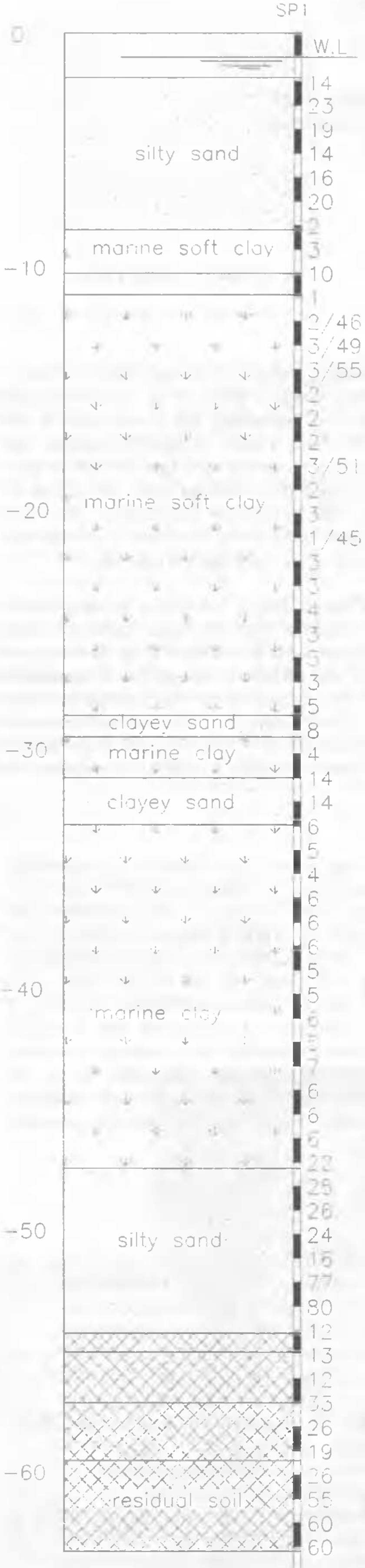


Figure 1. Subsoil profile.

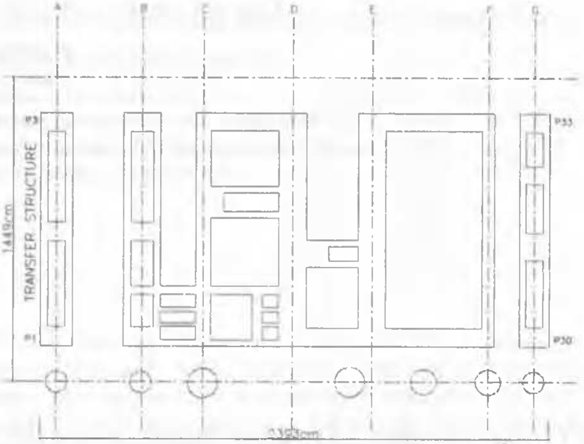


Figure 2. Plant of the transfer structure and piles.

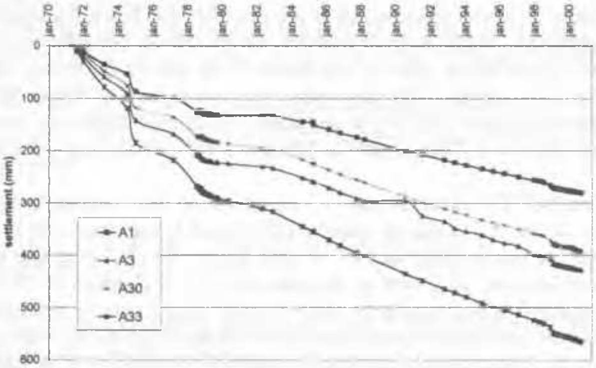
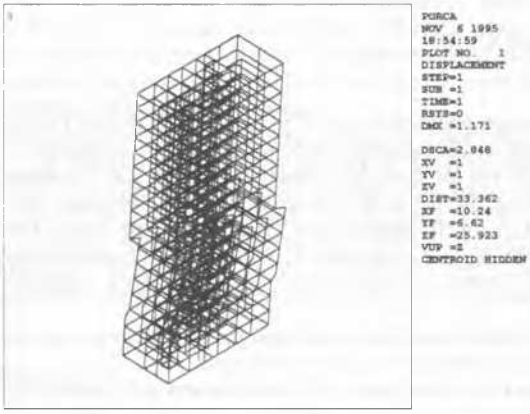


Figure 3. Recorded settlement (mm) vs. time (days)



```
PORCA
NOV  6 1995
18:54:59
PLOT NO.   1
DISPLACEMENT
STEP=1
SUB =1
TIME=1
RSYS=0
DMX =1.171

DSCA=2.048
XV  =1
YV  =1
ZV  =1
DIST=33.362
XF  =10.24
YF  =6.62
ZF  =25.923
VUP =Z
CENTROID HIDDEN
```

Figure 4. Structure collapse (numerical simulation).

permanent wind load about 150km/h on the right side of the build.

After this analysis, a vast measurement program was conducted on the structure in order to confirm the mathematical model. Through stress relief tests on several reinforcement steel bars it was possible to estimate the current stresses on them. The measured values were compared with the computed ones. The constitutive parameters of concrete were then reformulated in order to keep the stress data fitting within 10% error.

After that, a new analysis was performed, applying the settlement displacements with the measured average velocity. Figure 4 above shows the numerical collapse of the structure through a combined rupture of columns and beams. According to our mathematical model, the collapse would occur in 10 years.

3. RECOVERING PROJECT AND WORKS

The recovering project started with the implementation of sixteen bored concrete piles using bentonite slurry for excavation. Piles were excavated *55.0 m* long and with variable diameter between *1.0 m* and *1.4 m*. Seven transversal beams and many secondary longitudinal beams were built (figure 2, 5 and 7). Between the blocks of the piles and the transversal beams fourteen hydraulic jacks were placed with variable capacity between *5000 kN* and *9000 kN*. The beams have involved the columns transferring the loads for the new foundations. Figure 5 presents the solution of the underpinning. Due to the proximity of the piles in relation to the existent foundations large diameter steel casing 6m and 12m long, respectively in the less and more settled side, were driven before the excavation of the piles, in way to avoid the lateral deformation of the sand of the first layer, keeping its confinement.

The work began in November of 1998 and the piles were concluded in December of the same year. The blocks of the piles were built very close of the footings, arriving at a minimum distance of *1cm*. The beams were built in 9 months, due to countless constructive difficulties. The construction sequence of piles, blocks and beams were studied aiming to minimize additional differential settlements during works.

We remark that the building remained inhabited during the whole work. So all the utilities connections (hydraulic and electric installations) were made flexible to allow the normal function of the building during the plumbing work.

All the stages of the build rehabilitation were accompanied by means of level readings in all the columns. The settlements of the columns A1, A3, A30 and A33 during the construction work can be seen in figure 7. During the works the building continued to incline and reached *2.10 m* out of plumb line.

After the conclusion of the structure the jacks were placed in niches left between the blocks and the transversal beams, according to Figure 11. After a partial loading of the new foundations by the jacks, the footings were liberated by removing the sand under them through hydraulic erosion.

The inclination of the building was corrected in one month after almost three months, since October 13, 2000, of structural reaction analysis by applying different combination of forces in the jacks (figure 8). The jack corresponding to column A33 went up *80 cm* (figure 12) until the building becomes totally vertical again, as one can check in figure 10.

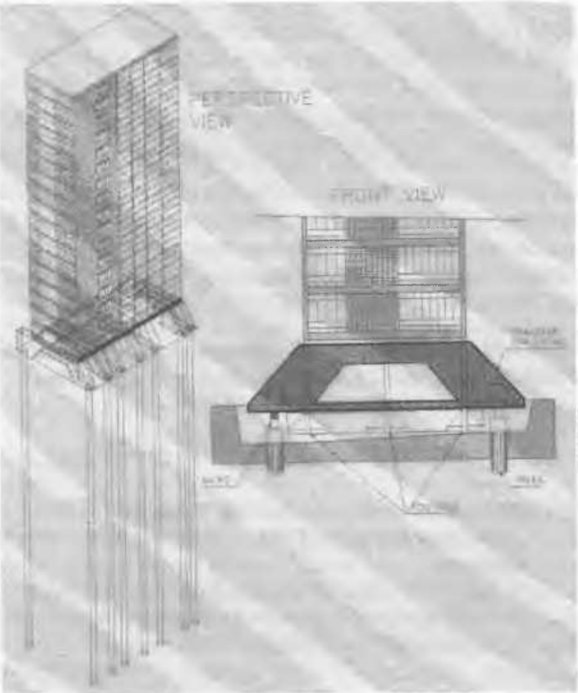


Figure 5. Applied solution.

Figure 6. Transfer structure at the first floor.

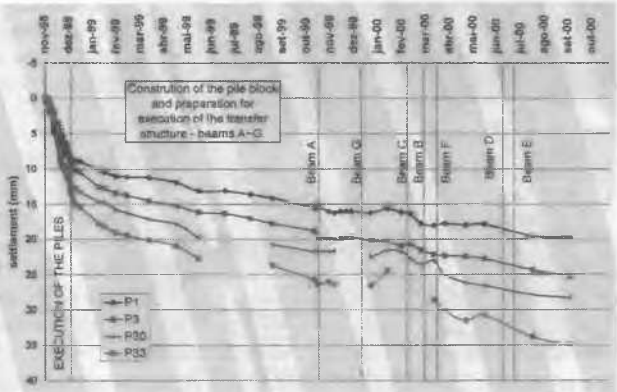


Figure 7. Recorded settlement vs. Time during the works.

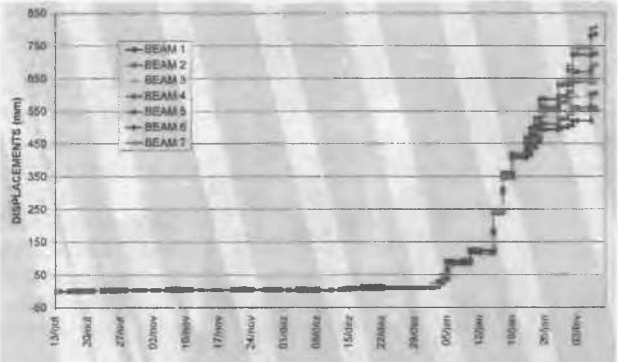


Figure 8. Applied displacements on the left side of the beams.

Figure 9. Tower A (at Right) in 1995.

Figure 10. Tower A after plumbing.

Figure 11. View of the jack at start time.

Figure 12. View of the block after jacking.

4 REFERENCES

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